PB82-205907

Industrial Reuse of Urban Stormwater

(U.S.) Municipal Environmental Research Lab.
Edison, NJ

Feb 81

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1. REPORT NO. EPA-600/J-81-547	2. Journal Article	3. PB82-205907
4. TITLE AND SUBTITLE INDUSTRIAL REUSE OF URBAN STORMWATER		5. REPORT DATE April 1982 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Richard Field and Chi-Yuan Fan		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Richard Field, U.S Environmental Protection Agency Storm and Combined Sewer Section, Edison, NJ 08837 Chi-Yuan Fan, Environmental Engineer, USEPA, Region II Municipal Section, New York, NY 10278		10. PROGRAM ELEMENT NO. 11. CONTRACT/GRANT NO.
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Environmental Research Laboratory Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268		13. TYPE OF REPORT AND PERIOD COVERED 14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES
Journal of The Environmental Engineering Division, ASCE, 107, (No. EE1); pp. 171-189, February 1981 issue.

16. ABSTRACT

As population and industry grow, water demand increases, and water supply becomes more of a problem. It has been estimated that the total gross water intake for all purposes in the United States will exceed the total available water supply of 650 billion gal/day (2.5 billion m^3/day) by the year 2000. Therefore, wastewater reclamation will become a more significant means of augmenting water supply.

Reclamation of municipal wastewater for industry, nonpotable domestic usages, and groundwater recharge has been practiced in the United States over the past several decades. In a 1971 United States Environmental Protection Agency supported (EPA) nationwide survey, it was estimated that current reuse of treated municipal wastewater for industrial water supply, irrigation, and groundwater recharge was 53.5 billion gal/yr, 77 billion gal/yr, and 12 billion gal/yr, respectively (see Ref. 1). It is reasonable to expect that the reuse of the treated wastewater for industrial cooling, nonpotable domestic water supply, and park and golf course irrigation will be substantially increased in the future.

Publications are available on the reuse of municipal effluent for potable and nonpotable water supply (see Refs. 2-4). However, very limited information is available on reuse and reclamation of urban stormwater. This paper examines current urban stormwater control and treatment technology leading to the feasibility of urban stormwater reuse for various purposes, including industrial cooling and process, and recreational water supply. The hydrologic and hydraulic aspects of urban stormwater systems are presented elsewhere and are not addressed here. A hypothetical case study illustrating the cost-effectiveness of reclaiming urban stormwater for complete industrial supply is presented.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Industrial water, Stormwater, Waste treatment, Water conservation, Water consumption, Water supplies	Control systems, Reclaimed water, Runoff, Urban areas, Water quality	

18. DISTRIBUTION STATEMENT	19. SECURITY CLASS *(This Report)*	21. NO. OF PAGES 12
	20. SECURITY CLASS *(This page)*	22. PRICE A02

EPA Form 2220-1 (Rev. 4-77) PREVIOUS E

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U.S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA 22161

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EPA-600/J-81-547
Journal Article

JOURNAL OF THE ENVIRONMENTAL ENGINEERING DIVISION

Industrial Reuse of Urban Stormwater

By Richard Field[1] and Chi-Yuan Fan[2]

Introduction

As population and industry grow, water demand increases, and water supply becomes more of a problem. It has been estimated that the total gross water intake for all purposes in the United States will exceed the total available water supply of 650 billion gal/day (2.5 billion m^3/day) by the year 2000. Therefore, wastewater reclamation will become a more significant means of augmenting water supply.

Reclamation of municipal wastewater for industry, nonpotable domestic usages, and groundwater recharge has been practiced in the United States over the past several decades. In a 1971 United States Environmental Protection Agency-supported (EPA) nationwide survey, it was estimated that current reuse of treated municipal wastewater for industrial water supply, irrigation, and groundwater recharge was 53.5 billion gal/yr, 77 billion gal/yr, and 12 billion gal/yr, respectively (see Ref. 1). It is reasonable to expect that the reuse of the treated wastewater for industrial cooling, nonpotable domestic water supply, and park and golf course irrigation will be substantially increased in the future.

Publications are available on the reuse of municipal effluent for potable and nonpotable water supply (see Refs. 2–4). However, very limited information is available on reuse and reclamation of urban stormwater. This paper examines current urban stormwater control and treatment technology leading to the feasibility of urban stormwater reuse for various purposes, including industrial cooling and process, irrigation, and recreational water supply. The hydrologic and hydraulic aspects of urban stormwater systems are presented elsewhere

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Note.—Discussion open until July 1, 1981. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on January 29, 1979. This paper is part of the Journal of the Environmental Engineering Division, Proceedings of the American Society of Civil Engineers,

and are not addressed here. A hypothetical case study illustrating the cost effectiveness of reclaiming urban stormwater for complete industrial supply is presented.

Quality and Quantity of Urban Stormwater and Combined Sewer Overflow

In recent years, a considerable number of characterization studies have been performed on urban stormwater. The reported stormwater quality parameters

TABLE 1.—Comparison of Quality of Storm Sewer Discharges for Various Cities[a] (5)

Type of wastewater, location, year, reference number (1)	BOD_5, in milligrams per liter		COD, in milligrams per liter		DO, in milligrams per liter	Suspended Solids, in milligrams per liter		Total Coliforms, in MPN/100 ml		Total Nitrogen, as N, in milligrams per liter	Total Phosphorus, as P, in milligrams per liter
	Average (2)	Range (3)	Average (4)	Range (5)	Average (6)	Average (7)	Range (8)	Average (9)	Range (10)	Average (11)	Average (12)
Typical untreated municipal	200	100-300	500	250-750	—	200	100-350	5×10^7	1×10^7-1×10^9	40	10
Typical treated municipal											
Primary effluent	135	70-200	330	165-500	—	80	40-120	2×10^7	5×10^6-5×10^8	35	7.5
Secondary effluent	25	15-45	55	25-80	—	15	10-30	1×10^3	1×10^2-1×10^4	30	5.0
Storm sewer discharges											
Ann Arbor, Mich., 1965 (2)	28	11-62	—	—	—	2,080	650-11,900	—	—	3.5	1.7
Castro Valley, Calif., 1971-1972 (14)	14	4-37	—	—	8.4	—	—	2×10^6	4×10^5-6×10^6	1.9[b]	—
Des Moines, Iowa, 1969 (6)	36	12-100	—	—	—	505	95-1,053	—	—	2.2	0.87
Durham, N.C., 1968 (1)	31	2-232	224	40-660	—	—	—	3×10^5	3×10^3-2×10^{6c}	—	0.18
Los Angeles, Calif., 1967-1968 (19)	9.4	—	—	—	6.9	1,013	—	—	3×10^3-2×10^6	—	—
Madison, Wisc., 1970-1971 (17)	—	—	—	—	—	81	10-1,000	—	—	4.8	1.1
New Orleans, La., 1967-1969[d] (16)	12	—	—	—	4.5	26	—	1×10^6	7×10^5-7×10^6	—	—
Roanoke, Va., 1969 (12)	7	—	—	—	—	30	—	—	—	—	—
Sacramento, Calif., 1968-1969 (37)	106	24-283	58	21-176	—	71	3-211	8×10^5	2×10^4-1×10^{7e}	—	—
Tulsa, Okla., 1968-1969 (33)	11	1-39	85	12-405	—	247	84-2,052	1×10^5	1×10^5-5×10^5	0.3-1.5[f]	0.2-1.2[f]
Washington, D.C., 1969 (5)	19	3-90	335	29-1,514	—	1,497	130-11,280	6×10^5	1×10^5-3×10^6	2.1	0.4

[a] Data presented here are for general comparisons only. Since different sampling methods, number of samples, and other procedures were used, the reader should consult the references before using the data for specific planning purposes.
[b] Only ammonia plus nitrate.
[c] Only fecal.
[d] Median values from one sampling station.
[e] Only organic (Kjeldahl) nitrogen.
[f] Only soluble orthophosphate.

TABLE 2.—Comparison of Quality of Combined Sewage for Various Cities[a] (5)

Type of wastewater, location, year, reference number (1)	BOD_5, in milligrams per liter		COD, in milligrams per liter		DO, in milligrams per liter	Suspended Solids, in milligrams per liter		Total Coliforms, in MPN/100 ml		Total Nitrogen, as N, in milligrams per liter	Total Phosphorus, as P, in milligrams per liter
	Average (2)	Range (3)	Average (4)	Range (5)	Average (6)	Average (7)	Range (8)	Average (9)	Range (10)	Average (11)	Average (12)
Typical untreated municipal	200	100-300	500	250-750	—	200	100-350	5×10^7	1×10^7-1×10^9	40	10
Typical treated municipal											
Primary effluent	135	70-200	330	165-500	—	80	40-120	2×10^7	5×10^6-5×10^8	35	7.5
Secondary effluent	25	15-45	55	25-80	—	15	10-30	1×10^3	1×10^2-1×10^4	30	5.0
Selected combined											
Atlanta, Ga., 1969 (31)	100	48-540	—	—	8.5	—	—	1×10^7	—	—	1.2[b]
Berkeley, Calif., 1968-1969 (34)[c]	60	18-300	200	20-600	—	100	40-150	—	—	—	—
Brooklyn, N.Y., 1972 (8)	180	86-428	—	—	—	1,031	132-8,759	—	—	—	1.2[b]
Bucyrus, Ohio, 1968-1969 (35)	120	11-560	400	13-920	—	470	20-2,440	1×10^7	2×10^5-5×10^7	13	3.5
Cincinnati, Ohio, 1970 (36)	200	80-380	250	190-410	—	1,100	500-1,800	—	—	—	—
Des Moines, Iowa, 1968-1969 (35)	115	29-138	—	—	—	295	159-1,166	—	—	12.7	11.6
Detroit, Mich., 1965 (2)	153	74-685	115	—	—	274	120-804	—	—	16.3[d]	4.9
Kenosha, Wisc., 1970 (18)	129	—	464	—	—	458	—	2×10^6	—	10.4[d]	5.9
Milwaukee, Wisc., 1969 (7)	55	26-182	177	118-765	—	244	113-848	—	2×10^5-3×10^7	3-24	0.8[b]
Northampton, England, 1960-1962 (22)	150	80-350	—	—	—	400	200-800	—	—	10[e]	—
Racine, Wisc., 1971 (18)	119	—	—	—	—	439	—	—	—	—	—
Roanoke, Va., 1969 (12)	115	—	—	—	—	78	—	7×10^7	—	—	—
Sacramento, Calif., 1968-1969 (37)	165	70-328	238	59-513	—	125	56-502	5×10^6	7×10^5-9×10^{7f}	—	—
San Francisco, Calif., 1969-1970 (3)	49	15-202	155	17-626	—	68	4-426	3×10^6	2×10^4-2×10^7	—	—
Washington, D.C., 1969 (5)	71	10-470	382	80-1,760	—	622	35-2,000	3×10^6	4×10^5-6×10^6	3.5	1.0

[a] Data presented here are for general comparisons only. Since different sampling methods, number of samples, and other procedures were used, the reader should consult the references before using the data for specific planning purposes.
[b] Only orthophosphate.
[c] Infiltrated sanitary sewer overflow.
[d] Only ammonia plus organic nitrogen (total Kjeldahl).
[e] Only ammonia.
[f] Only fecal.

vary considerably in concentration and mass. Storm runoff volume and flow rate vary greatly as well. These variations occur not only with time as the storm progresses, but also from location to location during the storm. Additional and significant influences on quality are attributable to drainage system configuration, antecedent dry weather period (allowing pollutant accumulation), and degree of imperviousness. Because of these multiple variations and the difficulties associated with representative sampling, relationships between cause and effect are obscured, even though a considerable amount of data are available. Table 1 (see also Ref. 5) summarizes a comparison of quality of stormwater discharges for various cities with that of untreated and treated municipal sewage.

The most obvious conclusion about the quality of urban stormwater is that it varies greatly from one metropolitan area to another. The data also indicate that the five-day biochemical oxygen demand (BOD_5) concentration is very close to that of secondary treatment plant effluent, while the suspended solids concentration is similar to that of raw sanitary wastewater.

A similar comparison for combined sewer overflow (CSO) quality is presented in Table 2 (see also Ref. 5). CSO is a mixture of various proportions of municipal sewage and storm runoff. For this reason, it would seem that the pollutant concentration of CSO would lie between that of the local sanitary sewage and urban storm runoff. The quality of sanitary sewage follows cyclical patterns over daily and weekly periods according to local demographic conditions. Therefore, the pollutant concentration of CSO may be affected significantly by the hour of storm occurrence. Marginally-sloped combined sewers encourage significant quantities of dry weather sanitary sewage solids deposition that accumulates during the low flow dry periods, and is subsequently flushed out by high storm flows creating "first flush" shock effects.

Control and Treatment Processes

Many unit processes designed for treatment of water and wastewater can be adopted to stormwater treatment. Because of the high volume and variability associated with storm and CSO, high-rate physical treatment units are considered to be advantageous over biological systems in many situations. Physical treatment alternatives have demonstrated the capability to handle high and variable flow rates and solids concentrations, whereas biological processes are more vulnerable to these variable flow conditions, and the relatively high concentration of nonbiodegradable solids in storm flow. Therefore, physical treatment systems are emphasized in this analysis.

Systems for urban stormwater reclamation may include storage and pretreatment, secondary treatment, and advanced treatment processes. Selection of treatment level is dependent on the reclaimed water usage. For example, water supply for irrigation requires a minimum degree of treatment, while that for boiler feed requires a high level. The various units for these basic processes are: (1) Control—storage or retention basin, and swirl regulator/concentrator; (2) pretreatment—bar/coarse screen, swirl concentrator, and sedimentation; (3) secondary treatment—combination of sedimentation, dissolved air flotation, or microscreening with biological treatment; and (4) advanced treatment—screening, filtration, and carbon adsorption.

The following sections describe treatment processes using test data obtained

from stormwater control demonstration projects supported by the EPA Office of Research and Development Urban Stormwater Control Technology Program (see Refs. 5–7).

TABLE 3.—Summary of Offline Storage Costs[a] (7)

Location (1)	Storage capacity, in millions of gallons (2)	Drainage area, in acres (3)	Capital cost,[d] in dollars (4)	Storage cost, in dollars per gallon (5)	Cost per acre, in dollars per acre (6)	Annual operation and maintenance cost, in dollars per year (7)
Akron, Ohio	1.1	188.5	455,700	0.41	2,420	2,900
Milwaukee, Wisc.						
Humboldt Avenue	3.9	570	1,774,000	0.45	3,110	51,100
Boston, Mass.						
Cottage Farm Detention and Chlorination Station[b]	1.3	15,600	6,495,000	5.00	416	80,000
Charles River Marginal Conduit Project	1.2	3,000	9,488,000	7.91	3,160	97,600
New York City, N.Y.						
Spring Creek Auxiliary Water Pollution Control Plant						
Storage	12.39	3,260	11,936,000	0.96	3,660	100,200
Sewer	13.00	—	—	—	—	—
Total	25.39	3,260	11,936,000	0.47	3,660	100,200
Chippewa Falls, Wisc.						
Storage	2.82	90	744,000	0.26	8,270	2,700
Treatment	—	—	189,000	—	2,100	8,000
Total	2.82	90	933,000	0.26	10,370	10,700
Chicago, Ill.						
Tunnels and pumping	2,998	240,000	870,000,000	0.29	3,630	—
Reservoirs	41,315	—	682,000,000	0.02	2,840	—
Total	44,313	240,000	1,552,000,000	0.04	6,470	—
Total storage treatment	—	—	1,001,000,000	—	4,170	—
Total	44,313	240,000	2,553,000,000	0.04	10,640	8,700,000
Sandusky, Ohio	0.36	14.86	520,000	1.44	35,000	6,200
Washington, D.C.	0.20	30.0	883,000	4.41	29,430	3,340
Columbus, Ohio						
Whittier Street	3.75	29,250[c]	6,144,000	1.64	210	—
Cambridge, Md.	0.25	20	320,000	1.28	16,000	14,400

[a] ENR 2000.
[b] Estimated values; facilities under design and construction.
[c] Estimated area.
[d] Land costs not included.
Note: 1 Mgal = 3,790 m^3; 1 acre = 0.405 ha; \$1/gal = \$264/m^3.

Storage.—Because storm flow is unsteady and intermittent, storage facilities are a necessary consideration for the system. Storage facilities may be constructed in-line or off-line. The in-line storage concept is based on using the excess trunk or interceptor sewer capacity for temporary detention of flow. Subsequently, the flow is released to the sewage (or stormwater) treatment plant.

Off-line storage is used to attenuate storm flow peaks, and it also provides some degree of treatment by enabling suspended solids to settle out. A multitude of storage facility types and configurations have been planned and constructed, including lagoons, and earthen-lined and unlined basins; open and covered concrete tanks; underground silos; deep tunnels; underwater bags; and void space storage (with overhead land used for other purposes). Concrete tanks are the most commonly-used storage facility for CSO, whereas earthen basins are usually employed for separate stormwater runoff storage.

TABLE 4.—Summary Data on Sedimentation Basins Combined with Storage Facilities in Operation (5)

Location of facility (1)	Size, in millions of gallons (2)	Type of storage facility (3)	Removal Efficiency, as a percentage		Type of solids removal equipment[a] (6)
			Suspended solids (4)	BOD_5 (5)	
Cottage Farm Detention and Chlorination Facility, Cambridge, Mass.	1.3	Covered concrete tanks	45	Erratic	Manual washdown
Chippewa Falls, Wisc.	2.8	Asphalt paved storage basin	18–70	22–74	Solids removal by street cleaners
Whittier Street	4.0	Open concrete tanks	15–45	15–35	Mechanical washdown
Alum Creek	0.9	Covered concrete tanks	NA[b]	NA	Mechanical washdown
Humboldt Ave., Milwaukee, Wisc.	4.0	Covered concrete tanks	NA	NA	Resuspension of solids by mixers
Spring Creek, Jamaica Bay, New York, N.Y.	10.0	Covered concrete tanks	NA	NA	Traveling bridge hydraulic mixers

[a]All facilities store solids during storm event and clean sedimentation basin when flows to the interceptor can handle the solid water and solids.
[b]NA = not available.
Note: 1 Mgal = 3,790 m^3.

The city of Boston has constructed the Cottage Farm Stormwater Detention and Chlorination Station which is designed to provide a minimum 10-min detention and chlorine contact to the 1-in-5-yr CSO rate of 233 mgd (883,000 m^3/day) (see Ref. 8); however the facility will provide complete capture or longer detention of the majority of flow coming from the more frequent storm events. The facility consists of bar screens and coarse screens, a pumping station, a hypochlorination facility, detention basins, fine screens, and an outfall. Flow is pumped to the detention tanks, each of which is gated to allow isolation for easier washdown and maintenance. Hypochlorite solution is fed to the pump

discharges. To reduce overflow solids carryover, detention tank effluent passes through fine mesh screens (<100-mesh openings) before discharge via the outfall. Screenings and settled solids are discharged to the downstream interceptor and finally conveyed to the domestic wastewater treatment plant. Because of the success of the Cottage Farm Detention and Chlorination Station, the Charles River marginal conduit project is now under construction. The purpose of this project is to improve water quality for recreational activities (see Ref. 9).

TABLE 5.—Pollutant Removal for Various Constituents by Sedimentation (10)

Pollutant (1)	Average removal, as a percentage (2)
Heavy metals[a]	
Copper	24.1
Chromium	32.3
Nickel	26.6
Zinc	27.2
Lead	30.6
Iron	16.6
Cadmium	38.8
Calcium	19.2
Magnesium	23.5
Sodium	18.5
Potassium	23.5
Mercury	8.4
Nitrogen[b]	
Ammonia	22.1
Organic	50.5
Total kjeldahl	38.4
Nitrate	15.4
Nitrite	0
Phosphorus[b]	
Total	22.2
Ortho	6.7
Other constituents[b]	
COD	34.4
TOC	21.3
Oil and grease[c]	11.9

[a]Average of 10 samples.
[b]Average of two–three samples.
[c]Average of six samples.

Other cities, i.e., Akron, Ohio; Milwaukee, Wisc.; Saginaw, Mich.; New York, N.Y.; Chippewa Falls, Wisc.; and Columbus, Ohio have similar facilities in operation for control of CSO (see Refs. 5, 7).

Costs of storage structures are highly dependent on location. Land, hydrogeological conditions, type structure and construction material, multipurpose usage, and esthetic appearance are primary considerations. Updated capital (excluding real estate), and operation and maintenance costs for storage facilities, are presented in Table 3 (see also Ref. 7).

Sedimentation.—The time-honored method for removing suspended solids is sedimentation. Storage installations also double as primary sedimentation facilities for flows that exceed storage capacity. Sedimentation has also been used for pretreatment and post-treatment, in addition to it being a dual benefit from storage. Removal efficiencies for CSO storage/sedimentation facilities are usually 30% for BOD_5 and 50% for suspended solids. Significant sedimentation demonstration and prototype project summaries are given in Table 4 (see also Ref. 5).

Removal of heavy metals, nitrogen, phosphorus, and other constituents by storage/settling has been reported and is summarized in Table 5 (see also Ref. 10).

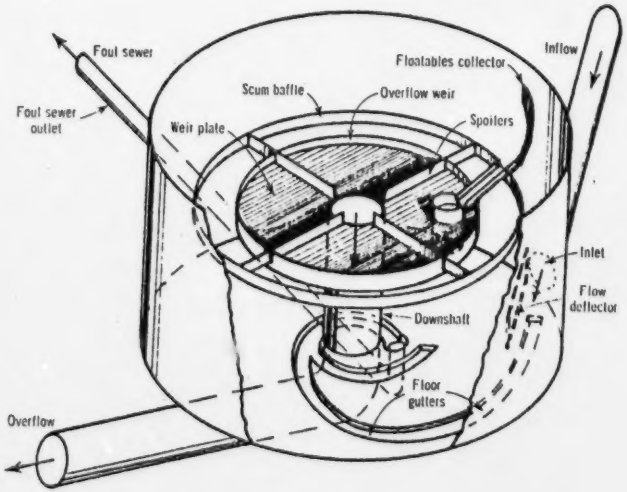

FIG. 1.—Isometric View of Swirl Regulator/Concentrator

Swirl Overflow Regulator/Concentrator.—The swirl regulator/concentrator achieves both quantity and quality control of storm flow simultaneously (see Ref. 11, 12). An isometric drawing of the device is depicted in Fig. 1. The principal mechanism for its dynamic solid-liquid separation ability is secondary fluid motion attained through long-path geometric flow patterns. The low flow concentrate is diverted to the sanitary sewer system, and relatively clear supernatant overflows the spillway (downshaft), and is diverted to the receiving water with or without subsequent treatment.

Based on a swirl CSO regulator/concentrator prototype demonstration in Syracuse, N.Y., suspended solids total mass removals were approx 50% (see Refs. 13, 14). In addition to the removal obtained by the physical splitting of flows, as with conventional regulators, a 20%–30% reduction in the suspended solids concentration is attributed to the swirl flow field's separation ability.

Swirl concentrators have been hydraulically modeled and are being demonstrated for various other purposes, including grit removal, and stormwater and erosion control (see Ref. 15).

Fine Screening.—Fine screens have been used for various applications including pretreatment, primary treatment, and final polishing. Because they allow relatively high throughput rates, screening units have been used for CSO. Screen openings

TABLE 6.—Characteristics of Various Types of Screens

Characteristic (1)	Microstrainer (2)	Drum screen (3)	Rotary fine screen (4)	Hydraulic sieve[a] (5)
Principal use	Main treatment	Pretreatment to other devices and main treatment	Pretreatment to other devices and main treatment	Pretreatment to other devices
Approximate removal efficiency, as a percentage				
BOD	50	15	15	—
Suspended solids	70	40	35	—
Land requirements, in square feet per millions of gallons per day	~15–20	15–20	24–62	20
Cost, in dollars per millions of gallons[b]	12,000	4,800	8,000	5,600
Can be used as a dry weather flow polishing device?	Yes	No	No	No
Automatic operation?	Possible, with controls	Possible, with controls	Possible, with controls	No controls needed
Able to treat highly varying flows?	Yes	Yes	Some limitation	Yes
Removes only particulate matter?	Yes	Yes	Yes	Yes
Requires special shutdown and startup regimes?	Yes	Some	Some	No
Screen life with continuous use?	7 yr–10 yr	10 yr	1,000 h	20 yr
Uses special solvents in backwash water?	No	No	Yes	No
High solids concentrate volume, as a percentage of total flow	0.5–1.0	0.5–1.0	10–20	<0.5

[a]Information on hydraulic sieves is limited. Formal study on treatment of combined sewer overflows is just beginning.
[b]Based on a 25-mgd (95,000 m^3/day) plant capacity, land costs not included.
Note: 1 sq ft/mgd = 4.65 ha/(m^3/day); \$1/mgd = \$0.000264/(m^3/day).

for microstrainers range from 23 μ–100 μ, and for the fine mesh type from 100 μ–1,700 μ. Removal efficiency is primarily a function of screen aperture dimension and flow suspended solids characteristics, and increases as the suspended solids grain size and concentrations increase and aperture dimension decreases. Solids removal results from two basic treatment mechanisms: (1) Direct straining by the screen; and (2) filtering of smaller particles by the mat, deposited on the screen surface by straining.

Microstrainers and fine mesh screens remove from 10%–99% of the suspended solids and from 10%–70% of the BOD_5. A summary of operation data is presented in Table 6 (see also Ref. 5).

Filtration.—In the area of physical treatment, filtration is one step finer than screening. Demonstrations made on filtration for treating urban stormwater and CSO have shown the most successful type to be the high-rate dual-media filter using anthracite coal over sand. A preliminary (420-μ) screening process was used upstream of the filters to extend the treatment run time before backwashing.

TABLE 7.—Typical Wet-Weather BOD and Suspended Solids Removals for Biological Treatment Processes[b]

Biological treatment process (1)	Expected Range of Pollutant Removal, as a percentage	
	BOD (2)	Suspended solids (3)
Contact stabilization	70–90	75–95
Trickling filters	65–85	65–85
Rotating biological contactors[a]	40–80	40–80

[a]Removal reflects flow ranges from 30 times–10 times dry weather flow.
[b]From Ref. 7.

It was found that suspended solids removal increased as influent suspended solids concentration increased, and decreased as hydraulic loading increased.

Removal efficiency for the filter unit was about 65% for suspended solids, 40% for BOD_5, and 60% for chemical oxygen demand (COD). The addition of polyelectrolyte increased the suspended solids removal to 94%, the BOD_5 removal to 65%, and the COD removal to 65%. The length of filtration run averaged 6 h at loading rates of 24 gpm/sq ft (1,400 m/day) (see Ref. 16).

Biological Treatment Processes.—Biological treatment of wastewater produces an effluent of high quality. These processes are generally categorized as secondary treatment processes capable of removing between 80% and 95% of the BOD_5 and suspended solids from sanitary sewage (see Ref. 7). When biological treatment processes are used for stormwater treatment, removal efficiencies are lower and controlled to a large degree by hydraulic and organic loading rates. As previously mentioned, most biological systems are extremely susceptible to overloading conditions and shock loads when compared to physical treatment processes. However, rotating biological contractors (RBC) have achieved high removals at flows 8 times–10 times dry weather design flows (see Ref. 17). Typical pollutant removals for contact stabilization, trickling filters, and RBC

are presented in Table 7 (see also Ref. 7) for wet-weather loading conditions. These processes include primary and final clarification.

Because of the limited ability of biological systems to handle fluctuating and high hydraulic and organic solids shock loads, storage facilities preceding the biological processes must be considered.

Activated Carbon Adsorption.—The role of the carbon adsorption step is to remove dilute soluble refractory, or residual organics, or both, from the wastewater. Carbon contacting systems generally employ granular activated carbon. The wastewater is passed either downward or upward through the carbon bed. An upflow expanded bed type is commonly used for wastewater treatment. The influent is introduced into the bottom of the column and flows up through the column at rates of 2 gpm/sq ft–8 gpm/sq ft (120 m/day–470 m/day). Exhausted carbon is removed from the bottom of the column and an equivalent quantity of fresh, or regenerated carbon, or both, is added to the top of the column. Periodical backwashing may be needed to remove excess biological growth and solids.

Organic demand removal capacity of carbon is about 0.5 lb·COD/lb·granular carbon (0.5 kg·COD/kg·granular carbon) (see Ref. 5). This is approximately equal to a requirement of 500 lb (225 kg) of activated carbon per million gallons of wastewater treated. When activated carbon is used for polishing or tertiary treatment, removal efficiencies for BOD_5 and suspended solids are above 95%.

A new development that employs powdered in lieu of granular activated carbon, which eliminates the need for the contact column, was demonstrated for the EPA for treating CSO, in Albany, N.Y. (see Ref. 18). This system consisted of a 0.10-mgd (380 m^3/day) trailer-mounted pilot plant where both powdered carbon and coagulants were added in a static mixing/reaction pipeline, and the resultant coagulated matter was flocculated downstream and separated by tube settlers.

Disinfection.—Disinfection of urban stormwater is generally practiced at treatment facilities to control pathogenic microorganisms. At most stormwater and CSO control installations, disinfection has been accomplished by applying conventional technology supplemented by high rate processes or on-site disinfectant generation. Studies have been made on high dosages, high-energy static and dynamic mixing, more rapid oxidants (e.g., ozone, ultraviolet radiation, and chlorine dioxide), or two-stage disinfection (using chlorine and chlorine dioxide), or both, to enhance the rate of bacterial kill. Successful attempts towards high-rate disinfection have been reported at the Philadelphia, Pa., Onondaga County, N.Y., and Monroe County, N.Y., demonstration sites (see Refs. 19–21).

The effectiveness of a disinfection system is influenced by contact time, type of disinfectant, dosage, mixing intensity, pH, protective suspended matter in the flow, and temperature. The contact time required for the process is inversely proportional to the concentration of the disinfectant and water temperature. For instance, it required five times as much contact time to obtain a 99% kill at 5° C, as it does to obtain the same kill at 30° C, at the same disinfectant dosage. Temperature is an important consideration here because storm runoff is more prone to temperature fluctuation.

Therefore, stormwater disinfection must be flexible and also capable of automatic operation to handle intermittent and varying storm flows.

The basic treatment and control methods indicated in the previous section have been examined essentially as singular processes; however system optimization should be considered to enhance overall treatment effectiveness and stormwater reuse potential. This section deals with integrated treatment systems to produce four levels of effluent for differing water quality classifications.

A typical flow diagram for several advanced physical-chemical treatment systems is shown in Fig. 2. Each treatment system produces a different degree of effluent water quality, except for sedimentation and fine screening high-rate filtration which result in similar treatment efficiencies. The flow diagram also indicates the water quality classifications associated with different water uses. Class AA is intended for high quality application, such as steam generation

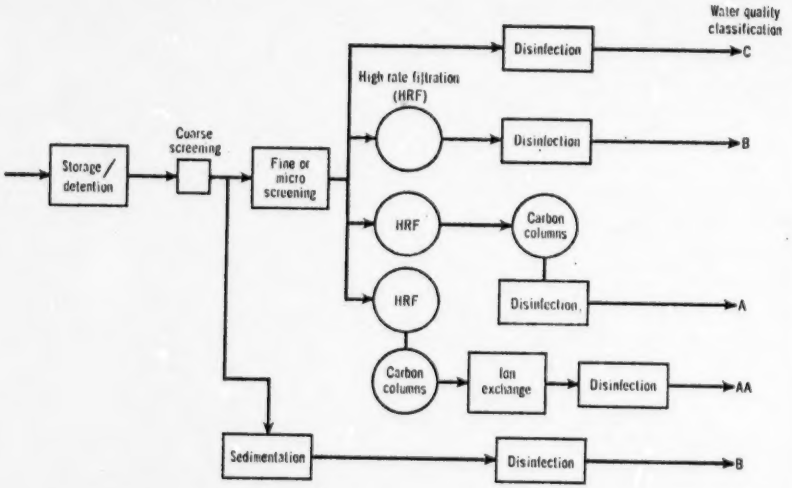

FIG. 2.—Typical Process Flow Diagram for Several Advanced Physical/Chemical Treatment Systems

boiler feed. Class A is intended for routine industrial process supply which has lower dissolved mineral removal requirements than Class AA. Class B can be used for industrial cooling and recreational water for fishing. Additional nutrient removal may be warranted. Finally, Class C is intended for lawn irrigation, fire protection, and esthetic ponds.

Table 8 (see also Ref. 22) indicates maximum concentrations of selected quality parameters. The water demands of each use must be evaluated with respect to the four aforementioned water quality levels defined in Table 8. The treatment systems described in the foregoing paragraphs are only intended to be representative of general conditions. Specific unit process selection and system design should only be determined after suitable investigation of the quality and quantity characteristics of the stormwater at the site and laboratory/pilot-scale evaluations.

HYPOTHETICAL CASE STUDY

A case study for a hypothetical industrial complex is presented to illustrate an engineering economic analysis of urban stormwater reclamation. Three water quality classification levels (A, B, and C, as shown in Table 8) were used to represent a range of industrial application. Three water supply source combinations, including reclaimed urban stormwater, or city water, or both, were considered as potential alternatives for this case study.

TABLE 8.—Maximum Concentration of Selected Pollutant by Reuse Category (22)

Constituent, in milligrams per liter, unless indicated (1)	REQUIRED WATER QUALITY, MAXIMUM CONCENTRATION[a] Water Quality Classification AA (2)	A (3)	B (4)	C (5)
Ammonia (NH_3)	0.5	0.5	0.5	0.5
Arsenic	0.01	0.05	0.05	0.05
Calcium	0.5	75.0	75.0	75.0
Chloride	50.0	250.0	250.0	250.0
Chromium (hexavalent)	0.05	0.05	0.05	0.05
Copper	1.0	1.0	1.5	1.5
Cyanide	0.01	0.2	0.2	0.2
Fluoride	1.5	3.0	3.0	3.0
Iron	0.1	0.3	0.3	0.3
Lead	0.05	0.1	0.1	0.1
Magnesium	0.5	150.0	150.0	150.0
Manganese	0.05	0.1	0.5	0.5
Nitrate (as NO_3)	45.0	50.0	50.0	50.0
Oxygen, dissolved (minimum)	5.0	5.0	4.0	4.0
Sulfate	50.0	200.0	400.0	400.0
Total solids	150.0	500.0	500.0	1,500.0
Zinc	5.0	15.0	15.0	15.0
Coliform (MPN/100 ml)	1	70	240	240
pH (units)	7.0	6.0	6.0	6.0
Color (units)	15	20	30	30
Turbidity (units)	0–3	3–8	8–15	15–20
Suspended solids	—	—	10.0[b]	30.0[b]
Phosphates	1.0	1.0	1.0	1.0

[a] Based upon maximum concentrations allowed by the United States Public Health Service, the World Health Organization, and the Water Quality Standards of the State of Maryland.
[b] Higher suspended solids are permitted by various water quality standards. Limit based on sediment control and water contact recreation.

1. Use of *city water* for lawn irrigation, potable, cooling makeup, and process supply.
2. Use of *reclaimed stormwater* for lawn irrigation, cooling makeup, process supply, and *city water* for potable supply.
3. Use of *reclaimed stormwater* for lawn irrigation and cooling makeup, and *city water* for potable and process supply.

Water Demands and Quality Needs.—The industrial complex is being established on a 100-acre (40.5-ha) tract 25% of which is used for open lawn and 75% for buildings, facilities, and roadways. Overall demand for water at the industrial site in the case study is expected to be about 8.0 mgd (30,000 m^3/day), based on assumed population (10,000 persons), lawn irrigation [4 in./week (100 mm/week) and eight months/yr], and in-plant cooling and processes. An additional intermittent demand of 9.11 mgd (34,500 m^3/day) for storage basin overflow disinfection was estimated based on long term hydrological analysis.

TABLE 9.—Water Demands and Quality Classifications (Hypothetical Case Study)

Use category (1)	Water Demand: In millions of gallons per day (cubic meters per day) (2)	Water Demand: In millions of gallons per year (millions of cubic meters per year) (3)	Water quality classification (4)
Lawn irrigation	0.39 (1,480)	9.40 (0.356)	C
Cooling makeup	5.0 (19,000)	1,500.0 (5.69)	B
Potable	0.89 (3,370)	267.0 (1.01)	City water
Process	1.61 (6,100)	483.0 (1.83)	A
Overflow disinfection	9.11 (34,500)	20.0 (0.076)	C

TABLE 10.—Water Quality Requirements (Hypothetical Case Study)

Parameter, in milligrams per liter (1)	Cooling makeup (2)	Process supply (3)
Total dissolved solids	1,500	500
Suspended solids	25–100	<10
pH[a]	6.9–7.5	7
Turbidity[b]	20	5
Calcium	50	50
Iron	0.5	0.3
BOD_5	25	2.0
COD	75	5
Sulfate	200	50

[a]In pH units.
[b]In Jackson turbidity units.

Table 9 summarizes the water demands and water quality classifications.

Table 10 presents typical water quality requirements for cooling tower makeup and process supply. These are assumed to be the hypothetical case study requirements.

Stormwater Reclamation System.—The stormwater reclamation system includes a storage reservoir and treatment facilities. The basic purpose of the system is industrial water supply, but the system also provides the multibenefits of enhanced drainage control, water pollution abatement, and improved local esthetics. In other words, the storage basin functions as a water supply reservoir, flow attenuator, pollutant load attenuator or separator, and esthetic pond. For pollution control, the reservoir should be designed and operated to prevent degradation of downstream receiving water quality. For water supply, the significant design criteria are industrial water demand rate, storage capacity, and the reliability with which the demand can be satisfied.

In studying storage requirements for the case study, the basin drainage locations (or catchment areas) were first analyzed to determine the industrial plant yield versus volume/capacity relationships, assuming 100% reliability over a representative 5-yr buildup period of stormwater inflow volume. A mass flow accumulation diagram (volumetric flow accumulation over a long time) with the known industrial process demand rate was used for this purpose.

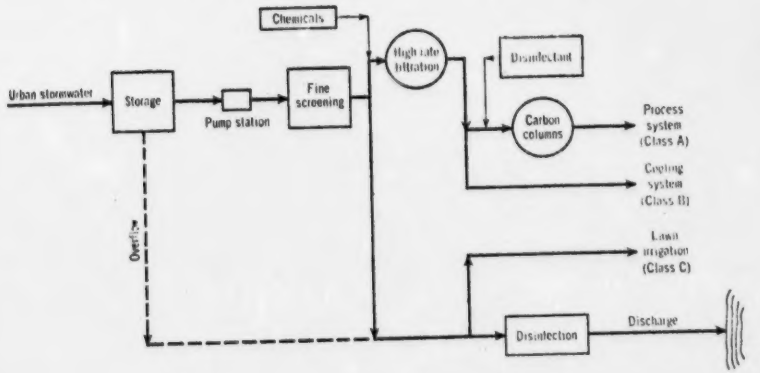

FIG. 3.—Proposed Treatment System Flow Diagram

Next, effluent water quality was analyzed in terms of system design and operating requirements. Finally, treatment process design criteria and capacities were developed which met all three plant water use (water classes A, B, and C) requirements. Detailed methodology and procedures for hydrological and process analyses are examined in other publications (see Refs. 5, 7, 22, 23).

Fine-mesh screening has had relatively wide use for storm flow treatment having been adopted for both the main process and for pretreatment in such processes as high-rate filtration and dissolved air flotation (see Refs. 5, 7, 15). High-rate filters hold promise for stormwater treatment based on large-scale pilot demonstrations (see Refs. 5, 7, 16, 24). Carbon adsorption is usually employed as a tertiary polishing step if a higher water quality is desired, and also has a demonstrated ability to treat stormwater (see Refs. 5, 18). The proposed treatment system contains these processes and is depicted in Fig. 3. The design flows and criteria of the unit processes are contained in Table 11.

TABLE 11.—Design Criteria for Storage-Treatment Facilities (Hypothetical Case Study)

Unit (1)	Design Flow Rate: In millions of gallons per day (2)	Design Flow Rate: In gallons per minute (3)	Design criteria (4)
Flow control/treatment			
Storage basin	200[a]		30 days
High-rate disinfection units	10	7,000	15 min.
Class C, B, & A			
Pumping station	7.2	5,000	—
Fine-mesh screens	7.2	5,000	50 gpm/sq ft
Class B & A			
High-rate filters	6.6	4,630	16 gpm/sq ft
Class A			
Carbon columns	1.6	1,130	5 gpm/sq ft 30 min.

[a]In millions of gallons; 1 mgd = 3,790 m³/day; 1 gpm/sq ft = 58.7 m/day.

TABLE 12.—Capital and Annual Costs for Urban Stormwater Treatment (Hypothetical Case Study)[a]

Process/equipment (1)	Capital cost, in dollars (2)	Annual Cost, in dollars: Amortization[b] (3)	O&M (4)	Total (5)
Flow control/treatment				
Storage basin	3,600,000	396,000	10,000	406,000
High-rate disinfection units	166,000	18,000	117,000	135,000
Class C, B, & A				
Pump station	975,000	107,000	90,000	197,000
Fine screening units	208,000	23,000	25,000	48,000
Class B & A				
High-rate filtration units	1,200,000	132,000	36,000	168,000
Chemical feed equipment and building	224,000	25,000	100,000	125,000
Class A				
Carbon Columns	1,300,000	143,000	90,000	233,000
Disinfectant feed system	120,000	13,000	100,000	113,000

[a]Unit cost from Ref. 25.
[b]n = 25 yr; i = 10%.

Cost Analysis.—The aforementioned design criteria were used to size the various units needed for the proposed treatment process, and then the capital and operating and maintenance (O&M) costs were estimated for the various alternatives. The capital costs include: (1) Structures, equipment, pumps, and piping; (2) engineering services, legal and administrative (estimated 20% of equipment and installation); and (3) land costs are $30,000/acre ($74,000/ha).

TABLE 13.—Estimated Total Annual Costs of Water Supply (Hypothetical Case Study)

Purpose (1)	Source (2)	Annual flow, in millions of gallons (3)	Unit cost, in dollars per thousand gallon (4)	Annual costs, in dollars (5)
		(*a*) Alternative 1		
Irrigation	City water	94.0	0.80[a]	75,000
Cooling	City water	1,500.0	1.05[b]	1,575,000
Process	City water	483.0	1.20[b]	9,580,000
Potable	City water	267.0	0.80[a]	214,000
Total				2,440,000
		(*b*) Alternative 2		
Irrigation	Class C (stormwater)	94.0	0.34	32,000
Cooling	Class B (stormwater)	1,500.0	0.48	720,000
Process	Class A (stormwater)	483.0	1.44	696,000
Potable	City water	267.0	0.80[a]	214,000
Total				1,662,000
		(*c*) Alternative 3		
Irrigation	Class C (stormwater)	94.0	0.34	32,000
Cooling	Class B (stormwater)	1,500.0	0.48	720,000
Process	City water	483.0	1.20[b]	580,000
Potable	City water	267.0	0.80[a]	214,000
Total				1,546,000

[a]Average water consumption charge rate in Middlesex County, N.J. (June 1978).
[b]Additional treatment required for control of biological growth, corrosion, or scaling, or both.
Note: 1 Mgal = 3,790 m³; $1/1,000 gal = $0.264/m³.

The capital costs were adjusted using the national average EPAWQO-Sewage Treatment Plant Index of 303.1 (June 1978).

Annual costs were based on a 25-yr amortization period at 10% interest and 300 days/yr of plant operation.

The proposed storage reservoir is of lined earthen construction and is 18 ft deep with a 2.5:1 interior slope, a 3:1 exterior slope, and a 16-ft top width

of levee. The reservoir is esthetically designed to harmonize with the local environment. Table 12 summarizes the capital, O&M, and total annual costs for the proposed stormwater reclamation treatment facilities using unit costs from Ref. 25.

Table 13 presents the summary of total annual costs of water supply for each alternative.

Based on the aforementioned analysis, Alternative 3, utilizing a combination of storm and city water supply, offers a clear savings of approx $900,000/yr over Alternative 1, which utilizes city water only. Even Alternative 2, which is mainly dependent on stormwater supply is $800,000/yr less than the exclusive city water supply alternative. Further savings are realized when the multipurposes of pollution control, drainage, and esthetics are considered.

Conclusions

This hypothetical case study provides evidence that the reclamation of urban stormwater for industrial subpotable water supply is technically feasible and economically attractive when compared to the city water source. In addition, other important benefits, such as reduction of pollutant discharges, drainage control, creation of recreational and esthetic ponds, groundwater recharge, and improvement and preservation of ecology in urban areas will be achieved.

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